KSCE Journal of Civil Engineering

Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests

--Manuscript Draft--

Manuscript Number:	KSCE-D-16-01811R1
Full Title:	Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests
Article Type:	Research Paper
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Funding Information:	National Natural Science Foundation of China (CN) (51479012) — Dr. Junjie Wang
	National Science and Technology Support Program of China (2015BAK09B01) — Dr. Junjie Wang
	Chongqing Science and Technology Commission of China (cstc2015jcyjBX0139) — Dr. Junjie Wang
Abstract:	The present study focuses on the coefficient of earth pressure at rest (K0) of a saturated crushed binary soil mixture. The mixture is artificially mixed by crushed sandstone and mudstone particles according to presupposed weight ratios and particle size distribution curves. The oedometer tests are performed to determine the coefficient K0. From test data, the values of coefficient K0, which range from 0.242 to 0.381, with a mean value of 0.300, are obtained. Just as other mechanical parameters of soils, the K0 value is affected by many factors. The effects of the properties of test specimen and material are discussed on the basis of the test data. Strong negative correlation between the values of K0 and initial dry bulk density of test specimen, one between the values of K0 and median particle size diameter of test material, and one between the values of K0 and gravel content by weight of test material, respectively, are observed from the test data. The mudstone particle content by weight of the mixture may also affect the value of K0. The effects of these factors on the K0 value may exhibit interlocking effect. Higher interlocking effect results in higher shear strength, and therefore results in lower K0 value.
Response to Reviewers:	The authors wish to thank the Editorial Office and the Reviewers for their insightful and

	constructive comments and advice on the manuscript. All these comments are very helpful to us for improvement of the quality of paper. The authors have taken full consideration of all these comments and made clarification and corrections as advised by the panel and reviewers. The detailed reply on the comments is listed in a fill named "Reply.doc".

Reply to All Comments on Manuscript No. KSCE-D-16-01811 Entitled "*Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests*"

by

Jun-Jie WANG, Yang YANG, Jiping BAI, Jian-Yun HAO, Tian-Long ZHAO

The authors wish to thank the Editorial Office and the Reviewers for their insightful and constructive comments and advice on the manuscript. All these comments are very helpful to us for improvement of the quality of paper. The authors have taken full consideration of all these comments and made clarification and corrections as advised by the panel and reviewers. The followings are our response to all comments:

For Reviewer #1:

- *"1. Page N. 2, line N.34: What do you mean by initial dry bulk density?"*

Reply: In the present manuscript, the initial dry bulk density is the dry bulk density of specimen after preparation prior to test. It is described at the end paragraph of "1. Introduction" section of the revised manuscript. In the abstract of page 2, it isn't explained because of limit of words.

- *"2. Page N. 2, line N. 35: What do you mean by median particle* *diameter?"*

Reply: In the present manuscript, the median particle size is the intermediate particle size diameter for which 50% by weight of the mixture smaller diameter. It is also described at the end paragraph of "1. Introduction" section of the revised manuscript. In the abstract of page 2, it isn't explained because of limit of words.

- *"3. Page N. 3, line N. 46: Are there any differences between coefficients of earth pressure at rest for total stress and effective stress parameters?"*

Reply: Thanks. The effective not total stress parameters were used to define the coefficient of earth pressure at rest by Jâky (1944). In our idea, for saturated soil, the difference between the coefficients of earth pressure at rest for total stress and effective stress parameters should be existed. The difference is:

For effective stress parameters: $K_{0e} = \dfrac{\sigma_h'}{\sigma_v'}$

For total stress parameters: $K_{0t} = \dfrac{\sigma_h}{\sigma_v}$

The two coefficients aren't equal, as $K_{0t} = \dfrac{\sigma_h}{\sigma_v} = \dfrac{\sigma_h' + u}{\sigma_v' + u} \neq \dfrac{\sigma_h'}{\sigma_v'} = K_{0e}$

- *"4. What is the range of coefficient of earth pressure at rest for well graded sand and poorly graded sand?"*

Reply: Thanks. The ranges of coefficient of earth pressure at rest for well graded sand and poorly graded sand are given in the first paragraph of "4. Results and Analyses" section of the revised manuscript, and as follows: "The K_0 value ranges from 0.242 to 0.381, with a mean value of 0.300. The K_0 value, for the test materials divided as SW or with the PSD curves #2, #3 and #4, ranges from 0.242 to 0.381, with a mean value of 0.301. And the K_0 value, for the test materials divided as SP or with the PSD curves #1 and #5, ranges from 0.253 to 0.346, with a mean value of 0.295."

- *"5. Page N. 4, line N. 67: What is the difference between bulk density and bulk dry density?"*

Reply: Thanks. In the position, the "bulk dry density" was replaced by "mass density" in the revised manuscript, because it means density only of soils.

- *"6. Page N. 6, line N. 107: what was the water content at saturated strength?"*

Reply: Thanks. Based on Chinese national standard GB/T 50266-2013 “Standard for test methods of engineering rock mass”, one of methods to saturate rock specimen is to soak the specimen in water for 48 hour. In the present study, the rock specimen was saturated using the method, but its water content at saturated state wasn’t determined.

- *“7. Page N. 9, line N. 176: what do you mean by in suit test methods?”*

Reply: Yes. This sentence is revised as follows: “The methods to measure the K_0 value may be divided into two types, test method in suit and one in laboratory. Test methods in suit for the measurements of K_0 value were grouped into three categories by Cai et al. (2011)……”

- *“8. Page N. 11, line N. 213: what was the criteria for choosing diameter and height of cylindrical specimen?”*

Reply: Thanks. This sentence is revised as follows: “The size of cylindrical test specimen was 61.8 mm in diameter and 40 mm in height, which was the specimen size suggested in Trade Standard of P. R. China SL237-028. (1999).”

Trade Standard of P. R. China SL237-028. (1999). Standard test methods for

at-rest earth pressure coefficient of soils. In Specification of Soil Test, The Ministry of Water Resources of P. R. China, Beijing, P. R. China (in Chinese).

- *“9. What is the effect of gravel size on the coefficient of earth pressure at rest condition? ”*

Reply: Thanks. In the present study, the effects of properties of test material on the coefficient of earth pressure at rest were investigated. The properties of test material include the median particle size diameter, gravel content by weight and mudstone particle content by weight, but no gravel size. The effect of gravel size may be similar to or different from one of the median particle size diameter. It isn’t confirmed because of absence of analyses. Based on the test data in the present study, the effect can’t be analyzed because the gravel sizes of test materials are the same 2.0-4.75 mm.

- *“10. What should be the suitable value of coefficient of earth pressure for filing material in foundation?”*

Reply: Thanks. Based on the definition of coefficient of earth pressure at rest, “in the numerical, the value of coefficient of earth pressure at rest (K_0) may be greater than one of coefficient of active earth pressure (K_a) but smaller than one of coefficient of passive earth pressure (K_p), i.e. $K_a<K_0<K_p$”.

It is difficult to give suitable value of K_0 because it is affected by too many factors.

The sentences in the quotation mark above are included in “1. Introduction” section of the revised manuscript.

For Reviewer #2:

- *“The manuscript entitled "Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests" reports an experimental study for the evaluation of the coefficient of earth pressure at rest (K0) and its correlation with material properties of artificially mixed soil. Despite the interesting work behind it, in the reviewer's opinion the manuscript cannot be accepted in its present form, mainly because of insufficient creativity, own contribution, and paper organization in the present form.”*

Reply: Thanks. Based on the useful comments from the three reviewers, the quality of the revised manuscript is improved largely. We sincerely hope that the manuscript is satisfied and accepted.

For Reviewer #3:

- *"The results of effects of median particle diameter in Fig.5, gravel content by weight in Fig. 6, and MP content by weight in Fig. 7 on coefficient of earth pressure at rest, respectively, are not so good. The authors are recommended to give some reasons and more discussions on these three figures. They are not so consistent compared with previous figures in the present study."*

Reply: Thanks for the good suggestion. Yes, the results shown in Figs. 5 and 6 are not so good. In the present study, the K_0 values shown the figures are calculated by fitting the experimental data of vertical effective pressure and horizontal effective pressure using the fitting straight line through coordinate origin expressed by Eq.(1), $K_0 = \frac{\sigma'_h}{\sigma'_v}$.

"Fig. 5 shows the relationship between the values of D_{50} and K_0. It is found that, in generally, the K_0 value varies with the D_{50} value, showing higher K_0 value for D_{50} value, except the points at D_{50}=0.35 mm." "The variation of K_0 value with the C_G value is shown in Fig.6. Compared with Fig.5, it is found that the effects of the C_G value on the coefficient K_0 are very similar to ones of the D_{50} value on the K_0 value. In generally, the K_0 values are increasing with the decrement of the C_G value, except the points at C_G=12%." "Based on the test data, the effects of the C_{MP} value on the K_0 value are plotted in Fig.7. As shown in Fig. 7, the variation of the K_0 value with increasing the C_{MP} value isn't monotonous, or can't expressed by a monotone function."

"The K_0 value, in Fig.5, at the point D_{50}=0.35 mm is greater than other points, and the K_0, in Fig.6, value at the point C_G=12% is also greater than other points. The two K_0 values at the point D_{50}=0.35 in Fig.5 and at the point C_G=12% in Fig.6 are the same for the test material with the PSD curve #4 and C_{PM}=80% and the test specimens with ρ_d=1.8 g/cm^3. From Fig.1, it is clear that the shape of the PSD curve #4 is different from other curves. The shape of PSD curve of test material may also affected the K_0 value, and further investigation on the issue may therefore be needed."

The sentences above were included in the revised manuscript.

Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests

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Abstract: The present study focuses on the coefficient of earth pressure at rest (K_0) of a saturated crushed binary soil mixture. The mixture is artificially mixed by crushed sandstone and mudstone particles according to presupposed weight ratios and particle size distribution curves. The oedometer tests are performed to determine the coefficient K_0. From test data, the values of coefficient K_0, which range from 0.242 to 0.381, with a mean value of 0.300, are obtained. Just as other mechanical parameters of soils, the K_0 value is affected by many factors. The effects of the properties of test specimen and material are discussed on the basis of the test data. Strong negative correlation between the values of K_0 and initial dry bulk density of test specimen, one between the values of K_0 and median particle size diameter of test material, and one between the values of K_0 and gravel content by weight of test material, respectively, are observed from the test data. The mudstone particle content by weight of the mixture may also affect the value of K_0. The effects of these factors on the K_0 value may exhibit interlocking effect. Higher interlocking effect results in higher shear strength, and therefore results in lower K_0 value.

Key Words: sandstone-mudstone particle mixture; coefficient of earth pressure at rest; oedometer test; property of test specimen; property of test material

1. Introduction

The evaluation on horizontal stress is always very important in many geotechnical engineering works such as slope (Sarma and Tan 2006), retaining wall or structure (Ahmad 2013) and pit (Kutschke and Vallejo 2012). There are three typical horizontal stresses or earth pressures in soil mechanics. They are the active earth pressure, earth pressure at rest and passive earth pressure. Many scholars investigated methods to calculate the coefficients of the three earth pressures under different conditions. In the numerical, the value of coefficient of earth pressure at rest (K_0) may be greater than one of coefficient of active earth pressure (K_a) but smaller than one of coefficient of passive earth pressure (K_p), i.e. $K_a<K_0<K_p$. The coefficient K_0 was defined as the ratio of horizontal effective pressure (σ'_h) to vertical effective pressure (σ'_v) in a soil that currently exists under the condition of zero horizontal deformation (Jâky 1944; Mesri and Hayat 1993; Mesri and Vardhanabhuti 2007). The coefficient K_0 is also given by:

$$K_0 = \frac{\sigma'_h}{\sigma'_v} \qquad (1)$$

It is relatively simple to compute the vertical effective stress, but the evaluation of horizontal effective stress is usually a complex task, because the value of coefficient K_0 depends on the precise geological and engineering stress history of soil deposit (Federico et al. 2009). Current approaches for evaluation of K_0 value are all empirical (Sivakumar et al. 2002). Although different methods to calculate the K_0 value were suggested by several investigators (Brooker and Ireland 1965; Fioravante et al. 1998; Michalowski 2005; Federico et al. 2008; Tong et al. 2013), the widely accepted method, which was suggested and adopted by Jâky (1948) and reported for example by Mayne and Kulhawy (1982), is written as:

$$K_0 = 1 - \sin\varphi' \tag{2}$$

where φ' stands for the effective internal friction angle of soil.

It is well known that the parameter φ' and other mechanical parameters of soils are usually affected by many factors such as soil type, particle size distribution, mass density, water content, stress state and oversize particle (Day 1989; Fragaszy et al. 1990; Fakhimi and Hosseinpour 2011; Xiao et al. 2014 and 2016; Zhao and Qiu 2016). The K_0 value should also be affected by some factors (Fukagawa and Ohta 1988). Based on published works related to the coefficient K_0, the factors include at least the soil type (Landva et al. 2000; Zhao et al. 2010; Talesnick 2012; Levenberg and Garg 2014), consolidation degree or state (Mayne and

Kulhawy 2003; Hanna and Al-Romhein 2008; Hayashi et al 2012; Grønbech et al. 2016), void ratio (Chu and Gan 2004,) stress state (Tian et al. 2009), pore water salinity (Yan and Chang 2015) and particle shape (Yun et al. 2015).

The mixture of crushed sandstone and mudstone particles, as a binary mixture (Vallejo, 2001), is often used as a main filling material in many geotechnical engineering works such as filling foundation, dam and wharf in Chongqing of China, where repeated strata of mudstone and sandstone are widely distributed. The repeated strata in Chongqing of China, which were mainly formed in Upper Triassic, Jurassic, and Lower Cretaceous periods, is about 2294 to 6440m in total thickness (CGMREDC 2002).

The evaluation on the earth pressure at rest of the binary mixture after filled is very important to the safety of engineering works, it should therefore be paid attentions by engineers and scholars. In the present study, in order to investigate the earth pressure at rest of the binary soil mixed artificially, the K_0 value was measured by laboratory experiments. Based on test data, the effects of some factors on the K_0 value were analyzed. The factors included the initial dry bulk density of test specimen (ρ_d, which is the dry bulk density of specimen after preparation prior to test), median particle size diameter (D_{50}, which is the intermediate particle size diameter for which 50% by weight of the mixture smaller), gravel content by weight of test material (C_G) and mudstone particle content by weight (C_{MP}) of test material.

2. Test Materials

In the present study, the test materials were the mixtures mixed artificially by crushed sandstone and mudstone particles (PS and PM). The preparation methods of the test materials include seven steps. The first step is to select lightly weathered sandstone and mudstone blocks in field. The second is to test uniaxial compressive strengths of the rock blocks in laboratory. The saturated strengths were 60.0-67.4 MPa (sandstone) and 8.3-15.0 MPa (mudstone). The third is to crush artificially the rock blocks into particles less than 4.75 mm in diameter. The fourth is to separate the crushed PS and PM, respectively, into 6 groups with different particle size diameter ranges 4.75-2.0 mm, 2.0-1.0 mm, 1.0-0.5 mm, 0.5-0.25 mm, 0.25-0.075 mm, and <0.075 mm (ASTM Standard D422-63 1998). The fifth is to mix the 6 PS groups according to the five presupposed particle size distribution (PSD) curves shown in Fig.1, and the 6 PM groups too according to the same PSD curves. The sixth is to mix the PS and PM with the same PSD curve together according to the weight ratio of PS to PM 2:8. And the last step is to mix the PS and PM mixtures with the PSD Curve #3 (see Fig.1) together according to the weight ratios of PS to PM 4:6, 6:4 and 8:2. There are altogether ten mixtures prepared. Five mixtures with the PSD Curves #1 to #5 and weight ratio of PS to PM 2:8 (i.e. C_{MP}=80%), and another five mixtures with the PSD Curve #3 and C_{MP} of 0%, 20%, 40%, 60% and 100%, respectively. The mixture with C_{MP}=0% is actually a crushed PS mixture only, and one with C_{MP}=100% is actually a crushed PM mixture only.

It is clear from Fig.1 that the maximum particle size diameter (D_{max}) of the test materials is the same, about 4.75 mm. In the present study, the size of cylindrical specimen described below was 61.8 mm in diameter and 40 mm in height. The problem of oversize particles, which may affect the properties of soils (Vallejo et al. 2004; Vallejo and Lobo-Guerrero 2012; Gutierrez et al. 2009), should be discussed. The term "oversize" refers to particles which are too large to be included in a particular test apparatus and is not associated with any fixed dimension (Fragaszy et al. 1990). In the present study, the ratios of D_{max} value to specimen diameter and height are 0.077 and 0.119, respectively. In order to eliminate or reduce the effects of oversize particles on shear strength of soils, ASTM Standard D3080/D3080M-11 (2011) recommends that the D_{max} value in a shear test must be no larger than one-tenth of the specimen width for square specimens or the specimen diameter for circular specimens. Based on the works of Fakhimi and Hosseinpour (2011), a D_{max} value to specimen width or diameter of 0.2 can be used in direct shear test with only minor loss in accuracy of the measured shear strength under the applied low normal stresses. For triaxial test, the D_{max} value should be limited to no more than one-sixth of the sample diameter (Vallejo and Lobo-Guerrero 2005; ASTM Standard D7181-11 2011). And in Chinese National Standard GB/T 50123-1999 (1999), for triaxial test with a specimen greater than 100 mm in diameter, the D_{max} value is limited to less than one-fifth of the sample diameter, and for one with a specimen smaller than 100 mm in diameter, the limit is one-tenth. For the oedometer test used in the present study, the limit on the D_{max} value wasn't reported in related published works or wasn't found by the authors. In the authors' idea, the limit for triaxial test (ASTM Standard D7181-11 2011) is also appropriate for the oedometer test. In the present tests, the limit of the D_{max} value is therefore 10.3 mm, about one-sixth of specimen diameter 61.8 mm. The limit of 10.3 mm is larger than the D_{max} value of the test materials, 4.75 mm, the problem of oversize particles can therefore be irrespective.

The properties of the test materials are listed in Table 1. It is clear that the D_{50} value of the test materials varies from 0.15 to 2.73 mm, the C_G value changes from 1% to 66%, and the C_{MP} value increases from 0% to 100%. The non-uniformity coefficient (C_u) and curvature coefficient (C_c) of PSD curves range from 4.52 to 25.56 and from 1.11 to 1.66, respectively. According to the unified soil classification system (USCS, ASTM Standard D2487 1985), the soil gradation for each tested material in the present study is divided as poorly graded sand (SP) or well graded sand (SW).

In order to investigate the effects of mass density of test specimen on the K_0 value, four values of ρ_d, 1.7, 1.8, 1.9 and 2.0 g/cm^3, are considered for the test material with the PSD Curve #3 and C_{MP}=80% (see Table 1).

3. Test Methods

In geotechnical engineering works, the evaluation on the coefficient K_0 is affected by series of uncertainties such as the mechanical properties of soils and calculation methods (Orr and Cherubini 2003). The experiment is therefore still a useful and reliable method to determine the K_0 value. The methods to measure the K_0 value may be divided into two types, test method in suit and one in laboratory. Test methods in suit for the measurements of K_0 value were grouped into three categories by Cai et al. (2011). They are direct measurements (for instance self-boring pressuremeter test), semi-direct measurements (for instance Marchetti's flat dilatometer test) and empirical correlations (for instance cone penetration test). Test methods in laboratory to determine the K_0 value of soils may mainly be divided into two types, i.e. triaxial and oedometer tests. In the triaxial test, which was performed in a special triaxial cell, the drainage of test specimen was allowed, the pore-water pressure was measured, and the axial load and cell pressure were adjusted to maintain a one-dimensional compression condition (Bishop 1958; Lefebvre and Poulin 1979; La Rochelle et al. 1981; Feda 1984). In the oedometer test, which was performed in a special oedometer cell, the soil test specimen was directly trimmed into highly polished stainless steel confining ring, and the pressure transducers were used to measure axial and lateral pressures (Abdelhamid and Krizek 1976; Holtz and Jamiolkowski 1985). Very recently, tactile pressure sensors used to measure the lateral pressure in centrifuge test were performed by Muszynski et al. (2016).

In the present study, the oedometer test in a special oedometer cell (Mesri and Hayat 1993) was used to measure the K_0 value of the test materials. Since by definition K_0 refers to a condition of null horizontal strains, some experimental difficulties arise in the measurement of horizontal effective stress, because almost all measurement devices need a (small) horizontal displacement (Lirer et al. 2011). In the oedometer test, some issues may affect the reliability of horizontal effective stress measurements. Two most important issues are the displacement of oedometric ring and side friction. The works of Lirer et al. (2011) and Lee et al. (2014) has testified that the error in the measurement of K_0 value caused by the oedometric ring deformability is very small. For the test condition adopted in the present study, it was confirmed that the range of induced strains was smaller than the limit value justifying the K_0 condition. The side friction may induce a variation of the axial stresses along the height of test specimen. The issue can be solved by measuring the axial stress at the mid height of test specimen, or by using the arithmetic average of top and bottom axial stress values. In the present oedometer test, the lateral pressure was measured using three pressure transducers installed on the mid height of side wall of an oedometer cell.

The size of cylindrical test specimen was 61.8 mm in diameter and 40 mm in height, which was the specimen size suggested by Chinese standard test methods for at-rest earth pressure coefficient of soils (Trade Standard of P. R. China SL237-028 1999). All the test specimens were prepared by wet tamping to produce a desired ρ_d value, with different test materials (see Table 1). The test specimen was saturated prior to test. During testing, the drainage of test specimen was always free. Four sequential pressures, 50, 100, 200 and 400 kPa, respectively, were applied on the top surface of test specimen step by step. Under each applied normal stress, the lateral pressure and vertical displacement (accuracy ±0.001 mm) were recorded every 6 sec until the displacement in an hour was less than 0.01 mm. The test method was selected from Chinese standard test methods for at-rest earth pressure coefficient of soils (Trade Standard of P. R. China SL237-028 1999) and American standard test methods for one-dimensional consolidation properties of soils (ASTM Standard D2435M-11 2011).

4. Results and Analyses

The relationship between the vertical and horizontal effective pressures for test specimens (4 replications) was analyzed. The vertical effective pressure was applied on the top surface of test specimen, and the horizontal effective pressure recorded while the vertical displacement of test specimen was less than 0.01 mm per hour. The typical relationship between the vertical and horizontal effective pressures is shown in Figs. 2 and 3. It is clear from the plots that, with the increment of the vertical effective pressure, the horizontal effective pressure is increasing along a fitting straight line through coordinate origin. The fitting straight line can also be expressed by Eq.(1). The slope of fitting straight line equals to the value of coefficient K_0. The values of K_0 and coefficient of determination R^2, which are calculated from the experimental data, are listed in Table 2. The K_0 value ranges from 0.242 to 0.381, with a mean value of 0.300. And the R^2 value ranges from 0.976 to 0.999, with a mean value of 0.989. The K_0 value, for the test materials divided as SW or with the PSD curves #2, #3 and #4, ranges from 0.242 to 0.381, with a mean value of 0.301. And the K_0 value, for the test materials divided as SP or with the PSD curves #1 and #5, ranges from 0.253 to 0.346, with a mean value of 0.295. Based on published works such as Lee et al. (2014), the K_0 value may be affected by applied vertical stress, but the effect is insignificant. The calculation of K_0 value (slope of fitting straight line between vertical and horizontal effective stresses as shown in Figs. 2 and 3) in the present study is therefore reasonable.

From the values of coefficient K_0 listed in Table 2, it is clear that the K_0 value may be affected by the properties of test specimen and material. In the present section, based on the experimental data, the effects of these factors are analyzed.

4.1. Initial bulk dry density of test specimen

The Case 1 listed in the first line of Table 1, for the test material with the PSD Curve #3 and C_{MP}=80%, and the test specimens with different values of ρ_d from 1.7 to 2.0 g/cm^3, was used to investigate the effects of the ρ_d value of test specimen on the K_0 value. The test data are listed in Table 2 and shown in Fig.2. Based on the test data, the relationship between the values of ρ_d and K_0 is shown in Fig.4. It is clear that the K_0 value varies with the ρ_d value, higher K_0 value for lower ρ_d value. The K_0 value ranges from 0.365 to 0.381 for ρ_d=1.7 g/cm^3, 0.313 to 0.329 for ρ_d=1.8 g/cm^3, 0.264 to 0.281 for ρ_d=1.9 g/cm^3 and 0.242 to 0.254 for ρ_d=2.0 g/cm^3. The tendency of decreasing K_0 value with increasing value of ρ_d or relative density D_r analyzed by Lee et al. (2014) is consistent with other test results for clean sands reported by Lee et al. (2013), sands containing fines reported by Mesri and Vardhanabhuti (2007), silty sands reported by Lee et al. (2014) and compacted sandy gravel reported by Lirer et al. (2011).

It is also clear from Fig.4 that, in generally, the variation of K_0 value with the increment of ρ_d value from 1.7 to 2.0 g/cm^3 may be expressed by a straight line. The fitting straight line is given by:

$$K_0 = -0.427\rho_d + 1.101 \qquad (R^2=0.964) \qquad (3)$$

This means that the increment of mass density of fills may reduce the value of earth pressure at rest. The larger ρ_d value of test specimen, the greater shear strength measured. The effects of the ρ_d value on the K_0 value may indirectly reflect the relationship between the coefficient K_0 and shear strength such as one expressed by Eq. (2) suggested by Jaky (1948), although the reliability of the Juky's equation was discussed by several scholars such as Lirer et al. (2011) and Lee et al. (2014).

The tendency of decreasing K_0 value with increasing ρ_d value may exhibit interlocking effects reported by Lee et al. (2014). Based on the works of Lee et al. (2014), higher interlocking effect may result in higher strength. Increasing ρ_d value may increase interlocking effect, and therefore decreases the K_0 value.

4.2. Median particle size diameter of test material

The Cases 2 to 5 and part of Case 1 listed in former five lines of Table 1, for the test materials with C_{MP}=80% and different PSD Curves #1 to #5, and the test specimens with ρ_d=1.8 g/cm^3, were used to investigate the effects of PSD on the K_0 value. The test data are also listed in Table 2. Based on the test data, Fig. 5 shows the relationship between the values of D_{50} and K_0. It is found that, in generally, the K_0 value varies with the D_{50} value, showing higher K_0 value for D_{50} value, except the points at D_{50}=0.35 mm. The values of K_0 range from 0.319 to 0.346 for D_{50}=0.15 mm, 0.358 to 0.381 for D_{50}=0.35 mm, 0.313 to 0.329 for D_{50}=0.83 mm, 0.271 to 0.292 for D_{50}=1.54 mm and 0.253 to 0.269 for D_{50}=2.73 mm. The tendency of decreasing K_0 value with increasing D_{50} value may also exhibit the interlocking effects reported by Lee et al. (2014). Wang et al. (2013b) reported that the angle of shearing resistance of an accumulation soil was generally increasing with the increment of the D_{50} value. Increasing the D_{50} value may also increase interlocking effect, and therefore decreases the K_0 value too.

It is also clear from Fig.5 that, in generally, the variation of K_0 value with the increment of the D_{50} value from 0.15 to 2.73 mm may be expressed by a fitting straight line. The straight line is given by:

$$K_0 = -0.036D_{50} + 0.353 \qquad (R^2=0.777) \tag{4}$$

This means that, in actual engineering works, increasing the D_{50} value of fills may also reduce the value of earth pressure at rest. The effects of the parameter D_{50} on the coefficient K_0 may be similar to one of the ρ_d value or shear strength analyzed above.

4.3. Gravel content by weight of test material

The Cases 2 to 5 and part of Case 1 listed in former five lines of Table 1, for the test materials with C_{MP}=80% and different PSD Curves #1 to #5, and the test specimens with ρ_d=1.8 g/cm^3, were also used to investigate the effects of the C_G value of test material on the coefficient K_0. The variation of K_0 value with the C_G value is shown in Fig.6. Compared with Fig.5, it is found that the effects of the C_G value on the coefficient K_0 are very similar to ones of the D_{50} value on the K_0 value. In generally, the K_0 values are increasing with the decrement of the C_G value, except the points at C_G=12%. The K_0 value, in Fig.5, at the point D_{50}=0.35 mm is greater than other points, and the K_0, in Fig.6, value at the point C_G=12% is also greater than other points. The two K_0 values at the point D_{50}=0.35 in Fig.5 and at the point C_G=12% in Fig.6 are the same for the test material with the PSD curve #4 and C_{MP}=80% and the test specimens with ρ_d=1.8 g/cm^3. From Fig.1, it is clear that the shape of the PSD curve

#4 is different from other curves. The shape of PSD curve of test material may also affected the K_0 value, and further investigation on the issue may therefore be needed.

The values of K_0 range from 0.319 to 0.346 for C_G=1%, 0.358 to 0.381 for C_G=12%, 0.313 to 0.329 for C_G=27%, 0.271 to 0.292 for C_G=40% and 0.253 to 0.269 for C_G=66%. The tendency of decreasing K_0 value with increasing C_G value may also exhibit interlocking effects. Based on the works of Wang et al. (2013b), the shear strength of an accumulation soil was generally increasing with increasing the C_G value of test material. Higher interlocking effect may be resulted from increasing the C_G value, and therefore decreases the K_0 value (Lee et al., 2014).

The relationship between the values of K_0 and C_G can be fitted by a straight line as shown in Fig.6. It is given by:

$$K_0 = -0.147C_G + 0.356 \qquad (R^2=0.725) \qquad (5)$$

This means that, in actual filling engineering works, increasing particle size or increasing content of large particles of fills may also be useful to reduce the value of earth pressure at rest.

4.4. Mudstone particle content by weight of test material

The Case 6 and part of Case 1 listed in the sixth and first lines of Table 1, for the test materials with the PSD Curve #3 and different C_{MP} values from 0% to 100%, and the test specimen with ρ_d=1.8 g/cm^3, were used to investigate the effects of the parameter C_{MP} on the coefficient K_0. The test data are also listed in Table 2 and shown in Fig.3. Based on the test data, the effects of the C_{MP} value on the K_0 value are plotted in Fig.7. As shown in Fig.7, the variation of the K_0 value with increasing the C_{MP} value isn't monotonous, or can't expressed by a monotone function. The values of K_0 range from 0.281 to 0.299 for C_{MP}=0%, 0.252 to 0.269 for C_{MP}=20%, 0.265 to 0.281 for C_{MP}=40%, 0.297 to 0.315 for C_{MP}=60%, 0.313 to 0.329 for C_{MP}=80% and 0.298 to 0.316 for C_{MP}=100%.

The variation of the K_0 value with the increment of the C_{MP} value may be fitted by a cubic curve (see Fig.7). The cubic curve is given by:

$$K_0 = -0.548C_{MP}^3 + 0.854C_{MP}^2 - 0.289C_{MP} + 0.289 \qquad (R^2=0.912) \qquad (6)$$

It is easily found from Fig.7 that there exist two specific values of C_{MP} at which the K_0 values are the minimum (at about C_{MP}=21.3%) and maximum (at about C_{MP}=82.6%), respectively. The effects of the C_{MP} value on the compaction behavior and particle crushing of the crushed sandstone-mudstone particle mixture were investigated by Wang et al. (2013a). According to the authors' works, due to the mixing of PM into PS, the maximum dry density of the mixture was increased, and its average relative breakage was decreased. The present study indicates that the effects of the mixing of PM into PS on the coefficient K_0 are also important. The effects of the C_{MP} value on the shear strength of the mixture were also investigated by Wang et al. (2016). The works of the authors indicated that the value of internal friction angle was decreasing with increasing the C_{MP} value. This is different a little from the relationship between the values of K_0 and C_{MP} shown in Fig.7 and expressed by Eq.(6). The effects of the C_{MP} value on the coefficient K_0 may be different from ones of the shear strength on the K_0 value expressed by Eq. (2), and may be more complex than ones of the parameters ρ_d of test specimen, and D_{50} and C_G of test material analyzed above. More investigation on the issue may be useful and interesting.

5. Summaries

Series of oedometer tests were performed to determine the K_0 value of a crushed sandstone-mudstone particle mixture. The mixture was frequently used as a fill to construct earth structures in many geotechnical engineering works in Chongqing of China. Based on the test data, the values of K_0 were obtained and analyzed. They range from 0.242 to 0.381 with a mean value of 0.300. The effects of several factors on the K_0 value were discussed by analyzing the test data. The analyzing results indicate that the K_0 value is generally reducing along a straight line with the increment of any of the initial bulk dry density of test specimen, median particle size diameter and gravel content by weight of test material. The variation of the K_0 value with increasing the mudstone particle content by weight of test material is well fitted by a cubic curve. The relationship between the values of K_0 and any of these factors may also exhibit the interlocking effects. Higher interlocking effect results in higher shear strength, and therefore results in lower K_0 value.

It was worth mentioning that the fitting Eqs. (3) to (6), obtained from the test data, exhibit the effects of several factors on the coefficient K_0, but their expressions may be changed for different materials.

Acknowledgements

The authors gratefully acknowledge the financial supports from the National Natural Science Foundation of China under Grant No. 51479012, the National Science and Technology Support Program of China under Grant No. 2015BAK09B01, and from the Chongqing Science & Technology Commission of China under Grant No. cstc2015jcyjBX0139, respectively.

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table 1

Table 1. Properties of test materials and specimens

Case No.	Properties of test materials							Properties of test specimens	
	PSD curve No.	Median particle size diameter D_{50} (mm)	Gravel content by weight C_G (%)	Non-uniformity coefficient C_u	Curvature coefficient C_c	Soil gradation	Mudstone particle content by weight C_{MP} (%)	Initial dry bulk density ρ_d (g/cm^3)	Number of test specimens
1	#3	0.83	27	25.56	1.16	SW	80	1.7, 1.8, 1.9, 2.0	4×4
2	#1	2.73	66	5.60	1.66	SP	80	1.8	4
3	#2	1.54	40	13.71	1.62	SW	80	1.8	4
4	#4	0.35	12	12.22	1.11	SW	80	1.8	4
5	#5	0.15	1	4.52	1.51	SP	80	1.8	4
6	#3	0.83	27	25.56	1.16	SW	0, 20, 40, 60, 100	1.8	5×4
Total									52

table 2

Table 2. Experimental values of coefficient K_0

No.	PSD Curve No.	Mudstone particle content by weight of test material C_{MP} (%)	Initial dry bulk density of test specimen ρ_d (g/cm^3)	Coefficient of earth pressure at rest K_0	R^2 of fitting straight line
1	#3	80	1.7	0.365	0.996
2	#3	80	1.7	0.375	0.991
3	#3	80	1.7	0.378	0.998
4	#3	80	1.7	0.381	0.989
5	#3	80	1.8	0.313	0.988
6	#3	80	1.8	0.320	0.991
7	#3	80	1.8	0.323	0.986
8	#3	80	1.8	0.329	0.993
9	#3	80	1.9	0.264	0.994
10	#3	80	1.9	0.274	0.997
11	#3	80	1.9	0.277	0.988
12	#3	80	1.9	0.281	0.999
13	#3	80	2.0	0.242	0.993
14	#3	80	2.0	0.244	0.991
15	#3	80	2.0	0.252	0.997
16	#3	80	2.0	0.254	0.986
17	#1	80	1.8	0.253	0.998
18	#1	80	1.8	0.262	0.977
19	#1	80	1.8	0.256	0.985
20	#1	80	1.8	0.269	0.991
21	#2	80	1.8	0.271	0.997
22	#2	80	1.8	0.281	0.991
23	#2	80	1.8	0.285	0.985
24	#2	80	1.8	0.292	0.976
25	#4	80	1.8	0.358	0.992
26	#4	80	1.8	0.367	0.985
27	#4	80	1.8	0.372	0.976
28	#4	80	1.8	0.381	0.988
29	#5	80	1.8	0.319	0.991
30	#5	80	1.8	0.326	0.996
31	#5	80	1.8	0.331	0.985
32	#5	80	1.8	0.346	0.979
33	#3	0	1.8	0.281	0.993
34	#3	0	1.8	0.299	0.991
35	#3	0	1.8	0.289	0.987

36	#3	0	1.8	0.287	0.998
37	#3	20	1.8	0.252	0.990
38	#3	20	1.8	0.269	0.978
39	#3	20	1.8	0.259	0.986
40	#3	20	1.8	0.264	0.997
41	#3	40	1.8	0.265	0.988
42	#3	40	1.8	0.281	0.991
43	#3	40	1.8	0.271	0.976
44	#3	40	1.8	0.275	0.995
45	#3	60	1.8	0.315	0.989
46	#3	60	1.8	0.297	0.982
47	#3	60	1.8	0.307	0.986
48	#3	60	1.8	0.310	0.994
49	#3	100	1.8	0.298	0.989
50	#3	100	1.8	0.316	0.991
51	#3	100	1.8	0.304	0.982
52	#3	100	1.8	0.306	0.988
Mean				0.300	0.989

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Fig. 1. PSD curves of test materials (Curve #*i* is the PSD curve for the *i*th test material)

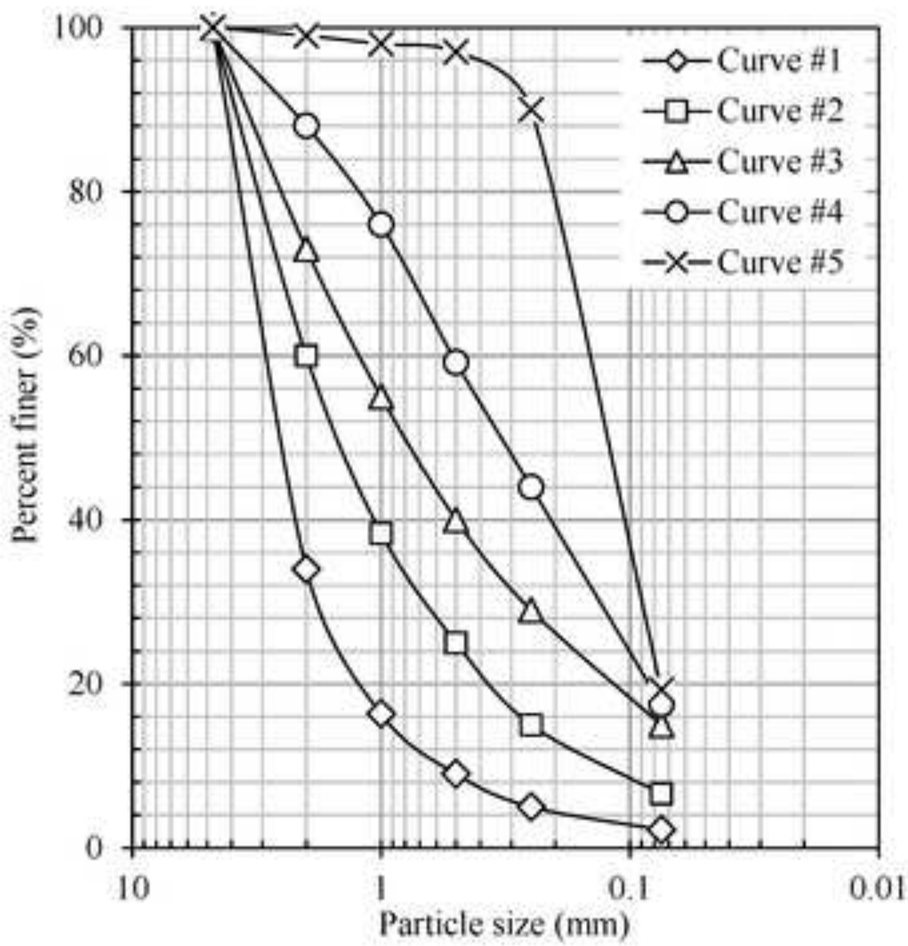

Fig. 2. Mean relationship between effective horizontal and vertical pressures for test material with PSD Curve #3 and C_{MP}=80% and test specimens with different ρ_d values

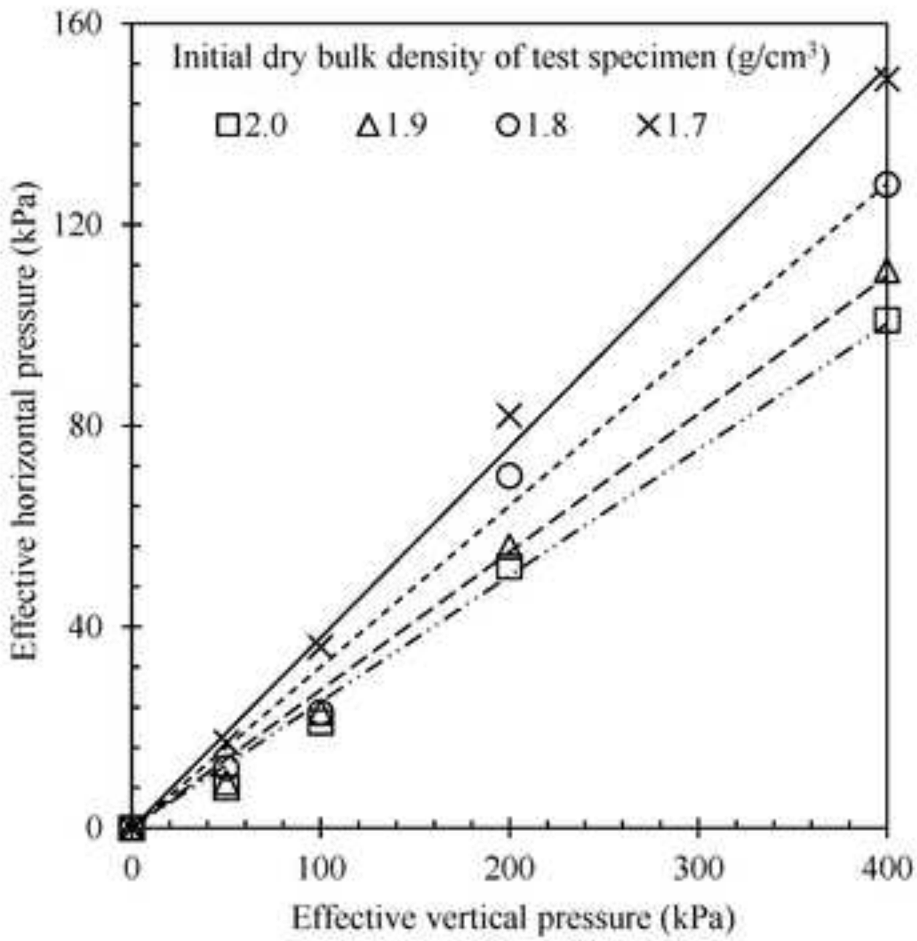

Fig. 3. Mean relationship between effective horizontal and vertical pressures for test materials with PSD Curve #3 and different C_{MP} values and specimens with ρ_d=1.8 g/cm^3

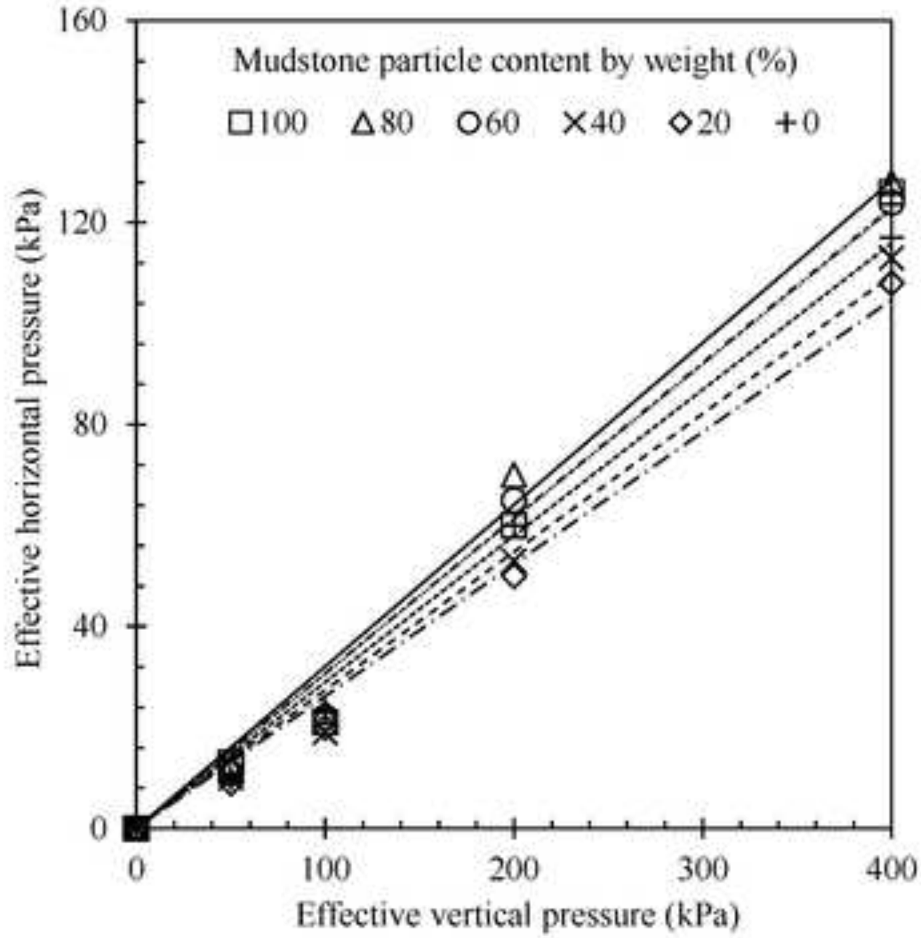

figure 4

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Fig. 4. Effects of initial dry bulk density of test specimen (ρ_d) on coefficient of earth pressure at rest (K_0)

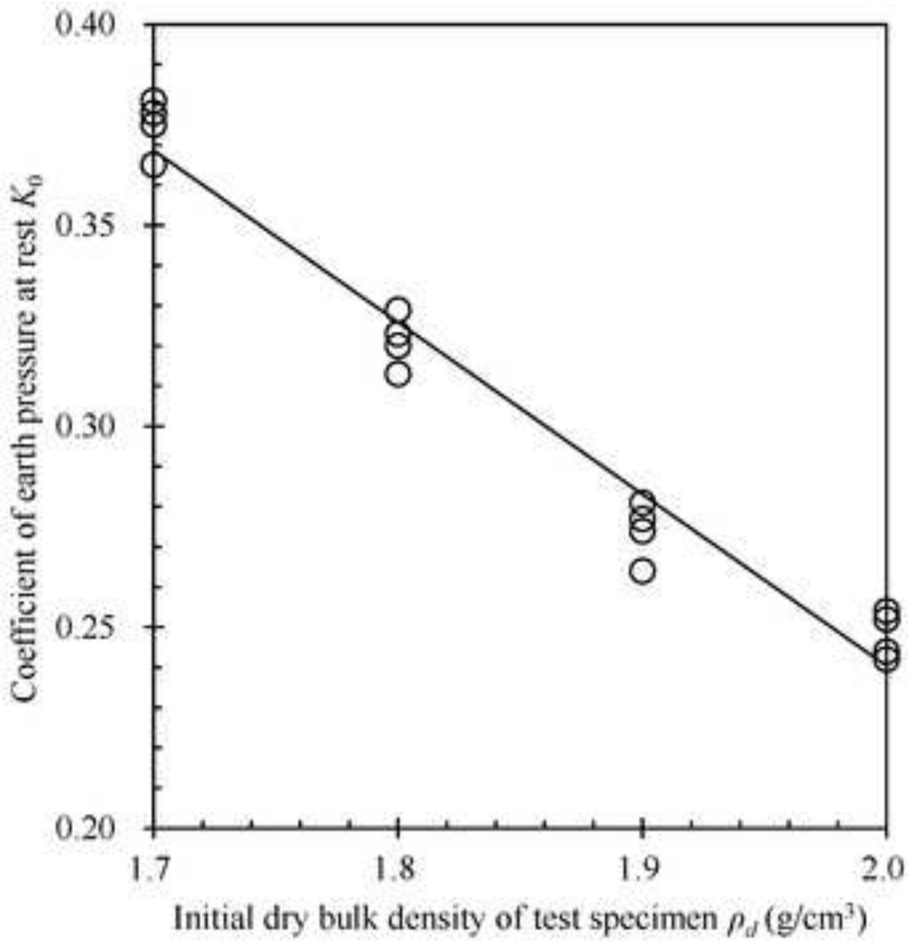

figure 5

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Fig. 5. Effects of median particle size diameter of test material (D_{50}) on coefficient of earth pressure at rest (K_0)

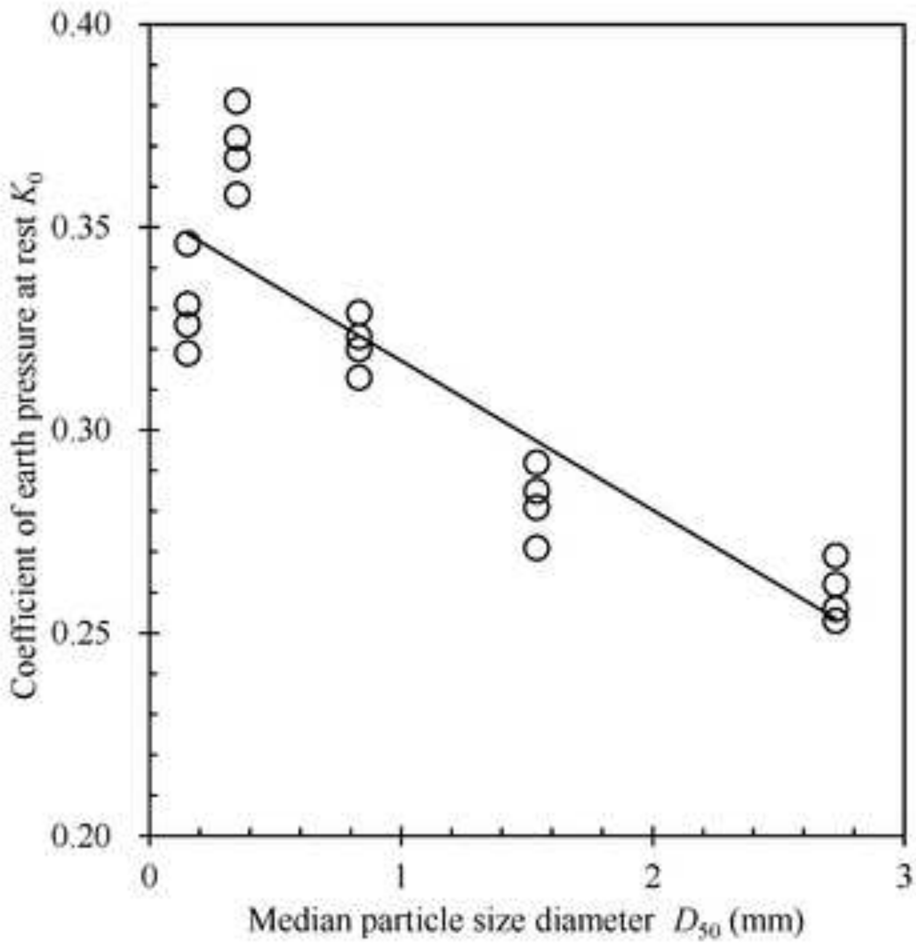

figure 6

Fig. 6. Effects of gravel content by weight of test material (C_G) on coefficient of earth pressure at rest (K_0)

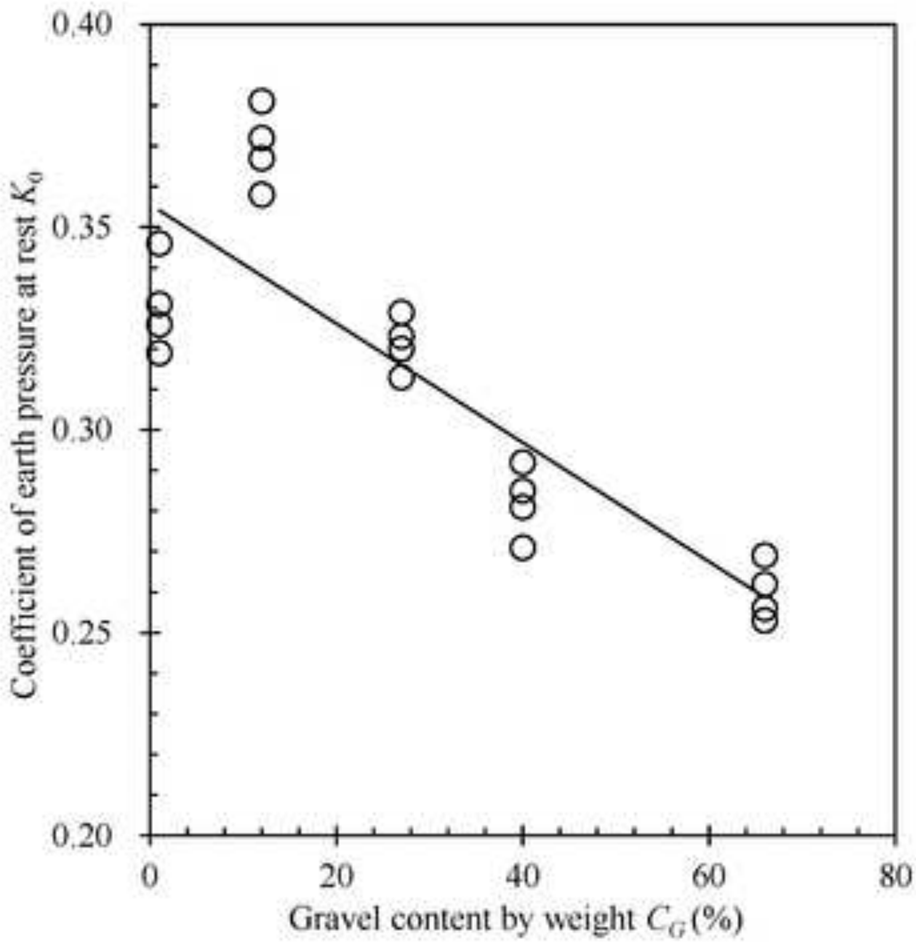

figure 7

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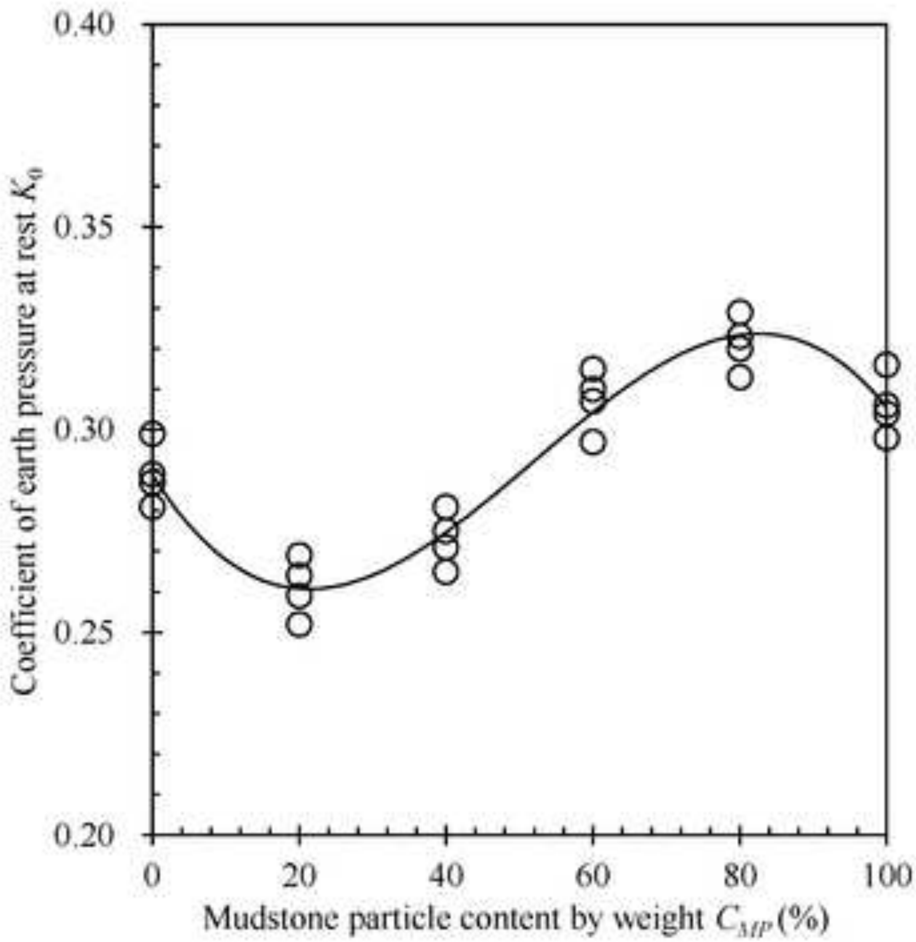
Fig. 7. Effects of mudstone particle content by weight of test material (C_{MP}) on coefficient of earth pressure at rest (K_0)



Dr. Jun-Jie WANG, Professor
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Mar. 5, 2017

Director
Journals Editorial Office
Dear Sir/Madam,

I am pleased to resubmit to you the manuscript No. KSCE-D-16-01811 Entitled "Coefficient of earth pressure at rest of a saturated artificially mixed soil from oedometer tests" for possible publication as a paper. The article has never been published elsewhere. All previously published work cited in the manuscript has been fully acknowledged.

In the revised manuscript, Professor Jiping BAI, from "School of Engineering, Faculty of Computing, Engineering and Science, University of South Wales, Treforest Campus, United Kingdom, CF371DL" is added as an author. The reason is that Professor Bai has revised the manuscript, especially improved English language of the revised manuscript, and gave some good suggestions about how to reply some comments and to improve the paper's quality. All authors of the revised manuscript, including Professor Bai of course, have approved the resubmitted version. I sincerely hope the change of authors can be approved by the Editorial Board.

Should you need to contact me, please use the above address or call me at (8623) 6289 6924. You may also contact me by fax at (8623) 6265 2841 or via e-mail at wangjunjiehhu@163.com.

I look forward to your early review comments.

Best regards.

Yours very sincerely,

Jun-Jie WANG

Professor